AD-A269 179

engineering
planning
surveying

1700 westland rd., cheyenne, wyoming 82001 (307) 637-6017

SANITARY SEWER MASTER PLAN

AND
PRELIMINARY DRAINAGE STUDY
FOR THE
SOUTH CHEYENNE WATER AND SEWER DISTRICT

JUL-16-1993 09:42 FROM HQ USAF CEA TO 97032749307 P.02

Air Force
Environmental Planning Division
(HQ USAF/CEVP)

Room 5B269
1260 Air Force Pentagon
Washington, DC 20330-1260

16 JUL 93

MEMORANDUM FOR DTIC (Acquisition)
(ATTN: Pat Mauby)

SUBJ: Distribution of USAF Planning Documents Forwarded on 1 JUL 93

All the documents forwarded to your organization on the subject date should be considered Approved for Public Release, distribution is unlimited (distribution statement A).

Jack Bush, GM-14
Mr. Jack Bush
Special Projects and Plans
703-697-2928
DSN 227-2928

JUL 16 '93 9:31 703 614 7572 PAGE.002

engineering
planning
surveying
a.v.i.p.c.

(307) 637-6017
1700 westland rd.

2-1270

March 29, 1985

Mr. Tom Bonds
Cheyenne/Laramie County Regional
Planning Office
2101 O'Neil Avenue
Cheyenne, Wyoming 82001

Dear Mr. Bonds:

We are pleased to submit this report to the South Cheyenne Water and Sewer District. This report includes the results of our analysis and evaluation of the District's sewer system. We also included recommendations for the improvements to the Districts system based on a maximum population of 15,000 people.

We would like to express our appreciation to the various officials and employees of the South Cheyenne Water and Sewer District, the Cheyenne/Laramie County Regional Planning Office, and the Cheyenne Board of Public Utilities for their cooperation and assistance during the preparation of this report.

Sincerely,

Eric Staab

Eric Staab, E.I.T.
AVI p.c.

Jim Voeller

James D. Voeller, P.E.
President, AVI, p.c.

ES:cmm

cheyenne, wyoming
82001

TABLE OF CONTENTS

PART A

SOUTH CHEYENNE WATER AND SEWER DISTRICT

SANITARY SEWER MASTER PLAN

I. SUMMARY AND CONCLUSIONS

This planning study is prepared for the South Cheyenne Water and Sewer District, and gives the necessary planning tools to evaluate the current and future conditions of their sewer system. The major steps in this study are as follows:

1. Mapping the existing sewer system.

2. Evaluating the theoretical capacity of existing collection lines.

3. Predicting the existing and the future sewage loads in these collection lines.

4. Recommending improvements to the existing system.

5. Recommending the improvements that will be necessary to handle future development.

6. Providing the District with a method of recording future development and projecting overloads in order to provide the appropriate relief sewers as they are needed.

Recommended relief sewers for existing and future populations are shown with the existing sewer system in the 7 maps at the end of this report.

To prevent a theoretical overload of the existing collection lines for the present service population, the following additions to the collection system are necessary in the immediate future.

3,980 ft. of 21" dia. pipe
6,350 ft. of 18" dia. pipe
3,500 ft. of 12" dia. pipe
(includes 108 ft. of 12" dia. cast iron pipe on Hwy. 85 crossing)

The estimated 1985 cost for these improvements is $800,000.00 not including road repair, movement of existing utilities and appurtenances, traffic control or geotechnical complications.

To prevent over loading in the collection lines for a future population of 15,000 residents, the following additions to the existing collection system will be necessary in the future:

2,632 ft. of 15" dia. pipe
5,410 ft. of 12" dia. pipe
(includes 108 ft. of 12" dia. cast iron pipe on Hwy. 85 crossing)
1,645 ft. of 10" dia. pipe
2,250 ft. of 8" dia. pipe

The estimated 1985 cost for these future improvements is $500,000.00, not including road repair, movement of existing utilities and appurtenances, traffic control, or geotechnical complications.

Funding for this Master Sewer Plan is provided by the Office of the Industrial Siting Administration, as part of the United States Department of Defense 801 Program, to assist in the impact assessment for the MX Missile Project.

The study uses existing as-built plans, existing studies, and mapping of the area. Field investigations and surveys to confirm existing sewer grades, physical conditions, and actual sewage loads are beyond the scope of this analysis. Private sewers and areas where no plans exist are not evaluated individually, but their impacts on the collection system are considered in the design loading calculations.

The study permits the Board to delay spending funds on the "immediate future" improvements until actual development demands it. If a building is added to an area, cross-referencing to the pertinent exhibits gives information about what manholes will be influenced, how much of a sewer line's potential flow will be utilized, and how much other sewer lines will be affected. Realizing these benefits, however, demands that an updated list of these exhibits be maintained as development takes place.

II. GENERAL DISCUSSION

Three classifications of sewage collection lines are discussed in this report. These classifications are:

1. Main (trunk) lines, which receive flow from various branch collectors and service laterals, and are 12" dia. pipe or larger. The District's trunk line consists of all of the 12" dia., 15" dia., 18" dia., and 21" dia. pipe in the sewage collection system.

2. Branch collection lines, which receive flow from individual services and more than one service lateral, and are less than 12" dia. pipe.

3. Service lateral lines, which collect flow from various individual services, and are smaller than 12" dia. pipe.

Branch collectors and service laterals that become overloaded affect relatively small areas, but overloaded main (trunk) lines tend to affect large areas, and eventually the entire service area. For this reason, discussions of South Cheyenne's trunk line has been divided into three (3) sections.

These three sections of trunk line meet in the sewer manhole at the intersection of Avenue C and Prosser Road. This is Manhole 12 (MH 12) and is shown (in duplication) on Exhibit 6 and on Exhibit 7. Two of these sections of trurkline flow into MH 12. The third section of pipe flows out of MH 12, and eventually delivers the entire sewage load to the Districts' sewage treatment facility.

The two trunk lines which flow into MH 12 collect most of the sewage generated in the system. The collection main draining into MH 12 at the west invert extends the length of Prosser Road and then south to Orchard Valley, receiving waste from the Holiday Inn, Orchard Valley, and other areas. The collection main into MH 12 at the south invert extends to the south on Avenue C to Nation Road, collects waste from Galaxie Estates, and provides trunk line service to the Districts' southern boundary.

These two mains cross Greeley Highway (State Highway 85-87) with 12" dia. collection mains. Both of these highway crossings will be insufficient for ultimate flows. The most northerly of the two on Prosser Road is overloaded now. The Nation Road collection main will be overloaded before the District reaches the design population of 15,000.

The combined flow from MH 12 to the treatment plant flows above safe limits now during peak loading conditions, and will need twice its existing capacity to provide adequate service to 15,000 residents.

The actual current residential population in the Districts' service area is approximately 6,500 residents. However, additional development has already been approved. Vacant lots, dwellings, and trailer spaces can also be occupied to increase the population within the District. For this reason, the approved population in the District is considerably greater than the actual population.

The recommended improvements are based on a "worst case" condition. Existing approved service population assumes that all dwellings, vacant lots, and commercial establishments now existing or approved for development within the Districts' service area are fully occupied. This includes approximately 8,650 residents, 70 commercial establishments, 2 elementary schools, Laramie County Community College, and the Holiday Inn. These services represent approximately 10,000 population equivalents serviced by the District, or 1.0 MGD for average daily flow (ADF), and 2.5 MGD for peak hourly flow (PHF). These flows will be experienced if no additional development within the District is allowed but all existing and approved lots are developed and fully occupied.

Future service population is based upon approximately 15,000 people and 120 commercial establishments. Total services in the area will represent 17,000 population equivalents, or 1.7 MGD for average daily flow and 4.25 MGD for peak hourly flow.

Even though the actual existing population (6,500 residents) is considerably less than that approved by the District (8,650 residents), the approved developments are being built and flows within the collection system will soon reach the values projected for existing approved population. If the recommendations for immediate improvements are not built before these flows occur, major maintenance problems may develop since the major collection lines for the entire district are in danger of being overloaded. Most serious is the 12" dia. collection main on Prosser, which will flow 77% full without any future development approval. The Cheyenne Board of Public Utilities criteria for maximum flow depth in sewer lines is 60% of full depth, to provide for ventilation, infiltration/inflow, and minor grease build-up.

The recommended future improvements accommodate the projected flow with 15,000 residences. However, in most cases these improvements represent the minimal amount of improvement. If the future District service population is expected to exceed 15,000 residences, consider increasing the recommendations to the next larger pipe size on a per improvement basis. For example, an additional 21" line parallel to the existing 21" collection main will provide for exactly the future population of 15,000. Running a 24" line parallel to the existing 21" will increase this capacity by an additional 3,600 population equivalents.

The study recommendations only represent improvements to the existing system. It assumes that all piping necessary to tie into the existing system will be the responsibility of individual developers, and are not needed. A proposed alignment for future developments is shown on the attached Master Plan Maps (Exhibits 1-7). Actual plans for these areas will need to be designed prior to any construction. The tie-in location with the existing system and its corresponding improvements need to be followed, but sewage routing to the designated points of impact are decisions necessarily left to future developers.

III. ANALYSIS/EVALUATION

The analysis/evaluation is divided into five sections. These sections are:

1. Determination of the maximum carrying capacity of existing sewer lines;

2a. Determination of the maximum potential flow in existing lines from existing population;

2b. Determining the maximum potential flow in existing lines from future population projections;

3a. Recommendations for relief sewers for the existing population;

3b. Recommendations for relief sewers for the future population;

4. Detailed analysis by area and manhole.

5. Recording future development and projection of overloads.

Section 1 is a technical discussion presenting the equations used to determine how much sewage a pipe can carry, based on the pipe size and its slope.

Section 2a. and Section 2b. describe how estimated quantities of sewage are determined. These quantities are based on the number and type of buildings in a given area, and how sewage is collected from these areas.

Section 3a. and Section 3b. summarize which pipes are too small. These Sections also summarize how to reduce the sewage flow through these pipes, and when the new sewer lines should be built to reduce these flows.

Section 4 is a detailed discussion of each area shown on the maps at the end of this report. The sewage flows in collection lines between manholes are also discussed. This information should be consulted when any new development in the District is considered.

Section 5 contains tables summarizing the information in this report. When new development in the District is approved, new values for existing population and flow into and between manholes should be recorded. The Section describes the method for revising this information as development exceeds existing population levels.

SECTION 1

DETERMINING THE MAXIMUM CARRYING CAPACITY

OF EXISTING SEWER LINES.

The factors that determine the maximum carrying capacity of gravity sewer lines are pipe size, slope, and maximum flow depth. Pipe size and slope for existing sewers are pre-determined physical characteristics. Maximum flow depth criteria is set at a maximum flow depth of 60% under peak hourly flows by the Cheyenne Board of Public Utilities.

Maximum carrying capacity is thus completely pre-determined and can be solved for by using:

$Q_{Full} = A\ (1.49/n)\ R^{2/3}\ S^{1/2}$ (Manning Equation)

Q_{Full} in cfs

A = x-sectional pipe area in $ft^2 = 3.14D^2/4$ (D=pipe dia.in ft.)

R = hydraulic radius in ft. = D/4 for circular sections

N = 0.013 (Recommended by the Clay Pipe Engineering Manual for all types of sewer pipe),

S = pipe slope in ft. per ft.,

and the relation Qmax/Qfull = 0.56 when dmax/dfull = 60% from the graph of Hydraulic Properties of circular sewers (Clay Pipe Engineering Manual).

The resulting equation is:

$Qmax = (0.56)\ 3.14D^2/4\ (1.49/0.013)\ (D/4)^{2/3}(S)^{1/2}$, Qmax in cfs.

Units for Q are in cfs. What we are interested in for planning purposes is the population that a particular line can service. Making the assumption that the average daily flow (ADF) is 100 gpcd and the peak hourly flow (PHF) is 2.5 times the ADF, PHF is then 250 gpcd. Using the conversion factor 1 MGD = 1.547 cfs, a PHF of 250 gpcd, the equation for Qmax above, and appropriate arithmetic yields

$$Q_{max} = 69.8\ (D)^{8/3}(S)^{1/2} \quad (1)$$

D = pipe diameter in inches

S = slope in ft per ft.

and Q_{max} in population equivalents (p.e.)

Equation 1 determines the maximum number of resident equivalents a sewer line of given size and slope can service. For instance, an 8" dia. pipe with slope = 0.004 ft/ft (D=8, S=0.004) results in Q_{max} = 1130 p.e. I.E., this line can service 1130 residents.

SECTION 2A

DETERMINING THE MAXIMUM POTENTIAL FLOW IN EXISTING LINES FROM EXISTING POPULATION.

The ASCE Manual on Engineering Practice No. 37 (Design and Construction of Sanitary and Storm Sewers) stresses the extreme importance of accurate population estimates in determining sewage flow. Because of the cited importance of accurate population estimates, this study is fortunate to have been preceeded by the technical report, Water System Analysis For The South Cheyenne Water and Sewer District, by ARIX, p.c. The ARIX study provided a means to determine existing and future population projections for residential, commercial, and institutional establishments and provided water use estimates for these establishments.

The correlation between water use and sewage disposed is rudimentary. The water system study by ARIX and this sanitary sewage study must correlate and be applied together if adequate service is to be supplied by the District.

The correlation between water use and sewage disposed is not a 1:1 ratio. Generally, only 60 to 80 percent of water demand becomes sewage.

One major difference between water supply and sewage demand in the District's service area is that while many homes (Orchard Valley in particular) maintain wells for water supply, most of these homes are connected to the District's sewer system. The ARIX study predicted a higher future demand of water per residence to account for wells being replaced by District service. This sewage study has assumed a constant sewage load of 100 gpcd.

The ARIX study showed the location of existing and future water taps for residences, apartments, trailer courts, institutions, and commercial establishments. It also showed the location of (7) seven trailer courts that were approved for construction. Two (2) of the seven have since been abandoned for immediate development. The other (5) five are in the process of development. These five are assumed to be fully populated for the purposes of this study. The two trailer courts that have been abandoned for immediate development are included in the study as future services.

Population densities per dwelling unit have been estimated in the ARIX study at:

Residence - 2.64 people per dwelling unit
Apartment - 2.50 people per dwelling unit
Mobile Home - 3.00 people per dwelling unit

These values are used to project population from the given number of dwelling units in the sewage study.

Population equivalents (p.e.) are the number of residents that an institution, commercial establishment, etc. are considered equal to in terms of average and peak flows. For instance, a trailer space is considered equal to 3 p.e. since the average trailer space in the District has 3 residents.

Population Equivalents (p.e.) for the following have been derived.

Holiday Inn: 700 p.e.
(This number is based upon the existing number of rooms being 347. This will allow 202 gpd per room at ADF).

Commercial Establishments: 13 p.e.
(This number is 80% of the peak hour demand for water supply demand projected for commercial establishments in the ARIX study).

Rossman School: 7 p.e.
Arp School: 6 p.e.
LCCC: 100 p.e.
(These numbers are based on a water use of 706 gpd, 590 gpd, and 150,062 gpd with no fluctuation as projected in the ARIX study. Note that allowances for fluctuation have been made for the sewage study).

Apartment buildings are assumed to contain (10) ten individual dwellings per building. (Results in slightly conservative estimating).

Population projections within the District's service area were determined by type and number of services depositing in various laterals, branch collectors and trunk lines. This study divided the District into 49 distinct areas with common influence on various service laterals, branch collectors and subsequent trunk lines. By assigning areas to services with common influence, flow projections were accumulated at various manholes throughout the District in population equivalents. The flow projections were then compared to the maximum capacity of the sewer lines, also in population equivalents. This comparison indicated which sewer lines were overtaxed.

SECTION 2B

DETERMINING THE MAXIMUM POTENTIAL FLOW IN EXISTING LINES FROM FUTURE POPULATION PROJECTIONS

The evaluation of maximum potential flows based on future population was similar to the evaluation of the existing flows. Population equivalents for each area were determined, based upon complete occupancy of all existing and future establishments in that area. Areas accumulated in branch collectors and trunk lines, and this accumulation was observed from manhole to manhole, and potential overloads to the existing system were determined.

The topography of individual areas affect how future loads will impact the existing system. Gravity sewers are generally most cost effective, and no sewage lift stations are proposed. By controlling where new development enters the existing system, overtaxed branch collectors and mains can be minimized. Strategic placement of relief sewers also minimizes the impact of future development.

Projections from the ARIX study were used to determine where new development is to occur, and what type of development it will be. The topography of the land to be developed was then analysed to determine in which direction(s) gravity sewer must flow to tie into the existing collection system. The existing system was then analysed to determine where existing lines will not be overloaded by future development, or where the impact from development to existing lines can be minimized. Upstream lines with insufficient capacity are sometimes by-passed when a line with sufficient capacity downstream is suitable for accepting flow from a development area by gravity flow.

SECTION 3A

RECOMMENDATIONS FOR RELIEF SEWERS FOR THE EXISTING POPULATION.

BRANCH COLLECTION LINES

At present only one branch collector is overloaded. This branch collector is 915' of 8" dia. sewer line between MH 24 and MH 23 on Exhibit 6. Its carrying capacity is 980 p.e. Its current load is 1123 p.e. and no future development is proposed. Its slope does not meet current minimum design standards. Tracy Long with the South Cheyenne Water and Sewer District indicated that this line requires maintenance 1 to 2 times yearly. This maintenance is not considered sufficient to justify the cost of upgrading, and since no future development is proposed, no recommendation is made to relieve or replace this branch collector. All manholes in Areas 46 and 47 (A-46 and A-47, Exhibit 6) should be inspected and repaired if necessary to minimize inflow. If roof drains and foundation drains exist, they should be eliminated. If maintenance increases to excessive amounts, existing 8" should be replaced by 10" dia. pipe. The existing manholes could be re-used if conditions warrant.

MAIN (TRUNK) LINE

Trunk line is defined by the Cheyenne Board of Public Utilities (supplement to Public Works Specifications not yet published) as 12" dia. or larger collection lines with various branch collectors and/or laterals contributing flow to it. The South Cheyenne Water and Sewer Districts trunk line consists of two trunk lines joining at MH 12 (at Avenue C and Prosser Road) and continuing as one to the sewage treatment plant.

At present the trunk line from MH 31 (Exhibit 6) on Prosser Road to MH 12 (Exhibit 6 and 7) on Prosser Road at Avenue C, is overloaded, and from MH 12 the trunk line is overloaded all the way to the sewage treatment plant. A relief sewer for this trunk line is recommended for immediate improvement.

This relief sewer is shown on Exhibit 6 and Exhibit 7. Exhibit 6 shows a 12" dia. relief main from MH 31 to MH 12 on Prosser Road to be installed parallel to the existing 12" dia. main. This installation will require 2 new oversized manholes to replace existing manholes, 10 new standard manholes, 3500 ft. of 12" dia. pipe, 108 ft. of road bore under Hwy. 85, and the intercepting of two existing lateral sewers.

Exhibit 7 shows an 18" dia. relief main to be installed from MH 12 to MH 3 down Prosser Road to the District boundary, then north along that boundary to MH 3. This installation will require 1 new oversized manhole, 5 new standard manholes, and 2000 ft. of 18" dia. pipe.

Exhibit 7 also shows an 18" dia. relief main parallel to the existing 18" dia. trunk line from MH 3 to MH 2 and from MH 2 to MH 1. This installation is needed now and will require 1 new oversized manhole, 10 new standard manholes, 4,350 ft. of 18" dia. pipe, and the intercepting of the separate sewer system (one lateral) from LCCC.

Also shown on Exhibit 7 is the start of the 21" dia. relief main parallel to the existing 21" dia. main that flows from MH 1 to the sewage treatment plant. This installation is needed now and will require 1 new oversized manhole, 9 new standard manholes, and 3980 ft. of 21" dia. pipe.

The total work necessary to relieve existing mains from flows that will occur when all establishments now approved are in use is:

1. The replacement of 5 existing manholes with new oversized manholes;

2. The installation of 34 new standard manholes;

3. The installation of 3,500 ft. of 12" dia. sewer pipe;

4. The installation of 6,350 ft. of 18" dia. sewer pipe; and

5. The installation of 3,980 ft. of 21" dia. sewer pipe.

These improvements are necessary to provide adequate service to the existing approved population. These improvements will also provide adequate service for all future development projected for this study period. Only the 15" dia. and 12" dia. trunk line from MH 12 (at Ave. C and Prosser Rd.) south, which is not overloaded from existing approved flows, will need relief main to handle future flows.

SECTION 3B

RECOMMENDATIONS FOR RELIEF SEWERS FOR FUTURE POPULATION

BRANCH COLLECTION LINES

Three branch collectors will be overloaded, to the point of needing relief, in the future as new development becomes approved and the District population approaches 15,000 residents. Exhibit 3 shows the first of these branch collectors to be discussed.

This branch collector, 8" dia. pipe, will become overloaded between MH 25 and the 12" dia. trunk line it flows into, 220 ft. away. By plugging the appropriate manhole inlets and pumping sewage while the 8" dia. pipe is replaced by 10" dia pipe, the existing manholes can be reused, provided they are in good condition. The installation of 220 ft. of 10" dia. pipe will carry future flows without overload, and is recommended as a future improvement.

The second branch collector that will be overloaded in the future is the branch collector between MH 38 (at the Holiday Inn) and MH 31 (where it flows into the 12" dia. trunk line on Prosser Road, shown on Exhibit 6). This branch collector, with one additional service lateral, provides service to the entire area south of Fox Farm Road and north of Prosser Road, west of Hwy. 85, and east of the District boundary. Also serviced is the Holiday Inn, which by itself generates 63 percent of the flow required to overload this branch collector. As development in this region occurs, this branch collector will become overloaded. Two relief sewers are recommended to amend the situation (both shown on Exhibit 6). The first is an 8" dia. relief sewer from MH 36 to MH 35 that will divert part of the flow from areas A43, A44, and A45 now flowing into the branch collector into the service lateral from MH 35 to MH 17. This service lateral will then change classification to become a branch collector itself, collecting flow from existing and future residential and commercial development and from the Holiday Inn. This improvement will require 1 new oversized manhole to replace an existing manhole, 2 new standard manholes, and 950 ft. of 8" dia. sewer pipe in order to provide relief to the existing branch collector downstream to MH 33 at Allison Road.

The existing branch collector from MH 33 to MH 31 will still be overloaded with future development, since significant development is projected. The branch collector is only 8" dia. The pipe slope in this run of pipe is slightly less than the minimum recommended design standard. An additional 8" dia. relief sewer is recommended to be run parallel to the existing 8" dia. line between MH 33 and MH 31, which will be sufficient to relieve the future flows. The requirements for this improvement are 1 new oversized manhole to replace an existing manhole, 3 new standard manholes, and 1,300 ft. of 8" dia. sewer pipe.

The last branch collector to be overloaded by future development is also shown on Exhibit 6. This branch collector will be overloaded from MH 9 to MH 5 as Areas 4 through 10 are developed. The 8" dia. pipe will need to be replaced with 10" dia. pipe to handle future flows, but the manholes themselves can be re-used, provided that they are still in usable condition. The requirements for this improvement are 1,425 ft. of 10" dia. pipe.

FUTURE IMPROVEMENTS TO MAIN TRUNK LINE

In the future, the trunk line from MH 19 at Terry Road and Nation Road (Exhibit 5) to MH 12 at Prosser Road and Avenue C (Exhibit 6) will be overloaded and a parallel relief main from MH 19 to MH 12 will need to be installed. This work will require 1 new oversized manhole to replace an existing manhole, 19 new standard manholes, 5,360 ft. of 12" dia. sewer pipe, 2,650 ft. of 15" dia. sewer pipe, and the intercepting of 3 branch collectors.

In summary, the future improvements to the existing system due to future development will consist of:

1. 3 new oversized manholes to replace existing manholes,
2. 24 new standard manholes,
3. 2,350 ft. of 8" dia. pipe,
4. 1,645 ft. of 10" dia. pipe,
5. 5,360 ft. of 12" dia. pipe, and
6. 2,650 ft. of 15" dia. pipe.

SECTION 4

DETAILED ANALYSIS BY AREA AND MANHOLE

Included in this Section are the quantities of sewage generated in each individual area, their point (s) of impact on the system, and the corresponding sewage loads that individual collection lines will carry. Also, are various comments, notes and considerations which have influenced the recommendations of this report. Particularities of individual areas and lengths of sewer pipe are described in detail, and when new developments are proposed, the District should review the appropriate discussion(s) in this section of the report.

This Section of the report is divided into seven exhibits, corresponding to the seven maps in the Appendix. An index for this section is also provided in Appendix A.

When any new development is proposed, after implementing this plan, the development should be located on one of the seven maps. Note the area number and the map number. Refer to the Summary Tables in Section 5 and the Index in Appendix A to determine which discussion(s) in this section would be beneficial to review.

EXHIBIT 1

1. Area 1 into MH 40.

Area 1 is serviced by 2 - 8" dia. branch collectors, with various 8" and 6" dia. laterals. The 8" dia. branch collector on E. Fox Farm Road has a carrying capacity of 1130 p.e., and the 8" dia. branch collector on Ave. D, has a carrying capacity of 980 p.e. These branch collectors converge at MH 40.

The existing sewage load in Area 1 is 750 p.e.
The future sewage load in Area 1 is 1173 p.e.

Since these loads are split between two branch collectors, these branch collectors will not be overloaded by future proposed development. No improvements are necessary in Area 1*.

Conversations with The South Cheyenne Water and Sewer District personnel indicated the only maintenance problems they have had in this area are tree roots in the lines on Turk Avenue. Maintenance has been seasonal.

* (Part of Area 1 is shown on Exhibit 7)

EXHIBIT 2

1. Area 3 into MH 39.

Area 3 is serviced by 1 branch collector and 1 service lateral which converge at MH 39. The carrying capacity of these collection lines vary, but are lightly loaded and no future expansion is proposed.

The existing sewage load in Area 3 is 188 p.e.
The future sewage load in Area 3 is 188 p.e.

No improvements are necessary in Area 3.

2. Area 45 into MH 38.

Area 45 is the Holiday Inn. Its service into MH 38 is a private collection line. There are 347 rooms in the Holiday Inn, and no future expansion is proposed.

The existing sewage load in Area 45 is 700 p.e.
The future sewage load in Area 45 is 700 p.e.

Improvements in Area 45 are not applicable.

EXHIBIT 3

1. Area 31 into MH 25,
   Area 30 into MH 26,
   MH 25 into MH 26.

Area 31 accumulates in two branch collectors at MH 25.

Area 30 accumulates in two service laterals that flow into the 12" dia. main line between MH 25 and MH 26.

The sewage line between MH 25 and MH 26 is all 12" dia. trunk line, except for 220 ft. of 8" branch collector servicing all of Area 31.

The existing sewage load in Area 31 is 597 p.e.
The future sewage load in Area 31 is 993 p.e.
The existing sewage load in Area 30 is 156 p.e.
The future sewage load in Area 30 is 156 p.e.

The carrying capacity of branch collectors and service laterals in Area 30 and Area 31 are sufficient to handle future sewage loads, except for 220' of 8" dia. branch collector between MH 25 and the 12" dia. trunk line. This branch collector has a carrying capacity of 980 p.e, and 993 p.e. are projected in the future. This line should be replaced by 220 ft. of 10" dia. pipe (carrying capacity of 1775 p.e.) before 383 additional population equivalents are developed in Area 31.

The 12" dia. trunk line between MH 25 and MH 26 is sufficient to carry all future sewage load. Its carrying capacity is 2885 p.e, while an existing sewage load of 753 p.e. and a future sewage load of 1149 p.e. accumulate at the west invert of MH 26.

Summary: Replace 220' - 8" dia. branch collector with 220' - 10" dia. branch collector when Area 31 develops by an additional 383 p.e.

EXHIBIT 4

1. Area 21 into MH 20,
   Area 22 into MH 21.

Area 22 accumulates in the 12" dia. trunk line flowing into MH 21.

Area 21 accumulates in the 8" dia. collection lateral flowing into MH 20.

The existing sewage load in Area 22 is 411 p.e.
The future sewage load in Area 22 is 1385 p.e.
The existing sewage load in Area 21 is 618 p.e.
The future sewage load in Area 21 is 768 p.e.

The 12" dia. trunk line has a carrying capacity of 2845 p.e., with an existing load of 411 p.e. and a future load of 1385 p.e. No improvements are necessary.

The 8" dia. service lateral has a carrying capacity of 1155 p.e., with an existing load of 618 p.e. and a future load of 768 p.e. No improvements are necessary.

Some flexibility is possible in the sewerage of these areas since neither the 8" dia. lateral or the 12" dia. trunk line are overloaded by future flows. Development in these areas should be recorded to make sure overloading is not permitted by indescriminate sewerage of new developments.

2. Area 49 (No development planned)

Area 49 is a large agriculturally zoned area and no future development is proposed. It is included only because it lies within the District's service area, and could influence the sewer system if unanticipated development occurs in this area.

EXHIBIT 5

1. MH 20 into MH 22,
   Area 24 into MH 22,
   Area 23 into MH 22,
   MH 21 into MH 22,
   MH 22 into MH 23.

Area 23 accumulates in the 12" dia. trunk line between MH 21 and MH 22.

Area 24 accumulates in the 8" dia. service lateral between MH 20 and MH 22.

The existing sewage load in Area 23 is 185 p.e.
The future sewage load in Area 23 is 434 p.e.
The existing sewage load in Area 24 is 85 p.e.
The future sewage load in Area 24 is 176 p.e.

The 12" dia trunk line between MH 21 and MH 22 has a carrying capacity of 2975 p.e., with an existing load of 596 p.e., and a future load of 1819 p.e. No improvements are necessary.

The 8" dia. service lateral between MH 20 and MH 22 has a carrying capacity of 1580 p.e., with an existing load of 703 p.e., and a future load of 944 p.e. No improvements are necessary.

The 12" dia. trunk line between MH 22 and MH 23 has a carrying capacity of 4965 p.e. The existing load is 1299 p.e. and the future load is 2763 p.e. No improvements are necessary.

2. Area 46 into MH 24,
   Area 47 into MH 23,
   MH 24 into MH 23.

The 8" dia. branch collector from MH 24 into MH 23 collects the flow from Area 46 at MH 24 and accumulates the flow from Area 47 from services and service laterals along this branch collector.

The existing sewage load in Area 46 is 1048 p.e.
The future sewage load in Area 46 is 1048 p.e.
The existing sewage load in Area 47 is 84 p.e.
The future sewage load in Area 47 is 84 p.e.

None of the service laterals by themselves are overloaded. However, the carrying capacity of the 8" dia. branch collector is 980 p.e. This branch collector is overloaded, and is carrying 1084 p.e. at MH 24 and 1132 p.e. by the time sewage enters MH 23. This line flows from between 63 and 66% full.

A discussion with South Cheyenne Water and Sewer District personnel supported this conclusion. He reported that this line plugs 1 to 2 times per year and is an annual maintenance problem.

Since no future development is proposed, the maintenance problems from this branch collector should not increase. If maintenance does become excessive it would be due to increased deterioration of services, laterals, and the branch collector, and a TV camera inspection should be done to determine if slip lining would provide sufficient relief of excessive flows. A decision as to when maintenance is excessive should be made by the District. If slip lining is determined to be inadequate, a 10" dia. line would be required to replace the existing 8" dia. branch collector.

Summary: No improvements to the existing system are recommended, but the District is advised to pay close attention to existing maintenance required in the branch collector between MH 24 and MH 23.

3. MH 23 into MH 19

The 430 ft. of 12" dia. trunk line between MH 23 and MH 19 has a carrying capacity of 2635 p.e. The existing sewage load is 2431 p.e. and the future sewage load is 3895 p.e. Additional capacity will be required when another 204 p.e. are approved upstream. An additional 12" dia. line is recommended to be built parallel to the existing 12" dia. trunk line (on the north). This parallel line will provide a total service of 5270 p.e. This installation includes 108 ft. of road bores under Highway 85.

Summary: Install an additional 430' of 12" dia. trunk line in the future, including 108 ft. under Highway 85.

4. MH 19 into MH 16,
Area 16 into MH 16,
MH 16 into MH 15.

The existing sewage load in Area 16 is 161 p.e.
The future sewage load in Area 16 is 424 p.e.

The 12" dia. trunk line between MH 19 and MH 16 has a carrying capacity of 2635 p.e. The existing flow accumulated in this line is 2592 p.e. and the future flow accumulated in this main is 4235 p.e. (an additional 84 p.e. flows into MH 16 through the 8" dia. service lateral on Avenue C in the future). This line will reach full capacity when an additional 43 p.e. are approved. An additional 12" dia. trunk line will provide a total of 5270 p.e. sewer service, and will be sufficient to handle all future loads.

The 12" dia. trunk line between MH 16 and MH 15 has a carrying capacity of 3940 p.e. The existing sewage load is 2592 p.e. and the future sewage load is 4319 p.e. An additional 12" dia. relief main will be required when 1348 p.e. are approved for development. If this relief main is not built when the relief main between MH 19 and MH 16 is built, one additional oversized manhole will be required (at MH 16) to route flow from the upstream relief main back into the existing main at MH 16. This alternative should be considered when determining construction phasing and priorities.

Summary: Install 2,255 ft. of 12" dia. relief main from MH 19 to MH 16 when an additional 43 p.e. are approved. Install an additional 1,400 ft. of 12" dia. relief main from MH 16 to MH 15 when an additional 1,348 p.e. are approved.

5. Area 25 into MH 18,
   Area 27 into MH 18.

From natural ground elevations and the plat for the Country West Subdivision provided by the District, Area 25 and Area 27 are to be sewered into MH 18.

The existing sewage load in Area 25 is 261 p.e.
The future sewage load in Area 25 is 378 p.e.
The existing sewage load in Area 27 is 0 p.e.
The future sewage load in Area 27 is 132 p.e.

The 8" dia. service laterals and branch collectors have minimum carrying capacities of 1130 p.e., and no improvements to the sewers in these areas will be necessary. The sum of these two areas represent the total sewage load at MH 18, 261 p.e. for existing and 510 p.e. for future sewage loads, respectively.

6. MH18 into MH 15,
   Area 15 into MH 15,
   MH 15 into MH 14.

The existing sewage load in Area 15 is 161 p.e.
The future sewage load in Area 15 is 363 p.e.

The carrying capacity of the 8" dia. branch collector from MH 18 to MH 15 is 980 p.e. The existing flow accumulated in this branch collector is 422 p.e. and the future sewage load is 873 p.e. No improvements to the existing system will be required to prevent overloading due to future development.

The carrying capacity of the 12" dia. trunk line between MH 15 and MH 14 is 3940 p.e. The existing sewage load is 3014 p.e. and the future load is 5192 p.e. An additional 12" relief main will be necessary when 926 additional p.e. are approved for development.

Summary: Install 1325 ft. of 12" dia. relief main in the future.

7. Area 14 into MH 14,
MH 14 into MH 13.

The existing sewage load in Area 14 is 285 p.e.
The future sewage load in Area 14 is 461 p.e.

The 8" dia. branch collector in Area 14 has carrying capacity of 1130 p.e. and will not require improvement to handle future development.

The 15" dia. trunk line between MH 14 and MH 13 has a carrying capacity of 4325 p.e. The existing sewage flow is 3299 p.e. and the future sewage load is 5653 p.e. An additional 15" relief will be necessary when 1026 p.e. are approved for development. This will increase the carrying capacity to 8650 p.e.

Conclusion: Install 1350 ft of 15" dia. in the future.

8. Area 13 into MH 13.

Area 13 is serviced by an 8" dia. branch collector on College Drive with a carrying capacity of 1585 p.e.

The existing sewage load in Area 13 is 160 p.e.
The future sewage load in Area 13 is 318 p.e.

This branch collector is lightly loaded and no improvements will be necessary to handle future overloads. The accumulated flow at MH 13 is 3459 p.e. for existing loads and 5971 p.e. for future loads.

9. Area 26 into MH 26,
Area 29 into MH 27,
MH 26 into MH 27.

Area 26 is serviced by 2 branch collectors. The branch collector on Citrus St. is the main collector for this area and has a carrying capacity of 1130 p.e. Elevation data for the branch collector in the Milzato Subdivision was not available, but this line is very lightly loaded.

Area 29 is serviced by several service laterals accumulating in the 12" dia. trunk line between MH 26 and MH 27. These service laterals have a minimum carrying capacities of 1130 p.e.

The existing sewage load in Area 26 is 300 p.e.
The future sewage load in Area 26 is 300 p.e.
The existing sewage load in Area 29 is 232 p.e.
The future sewage load in Area 29 is 232 p.e.

No service laterals in these areas are overloaded or will be overloaded in the future.

The 12" dia. trunk line between MH 26 and MH 27 has a carrying capacity of 3260 p.e. Its existing sewage load is 1285 p.e. and its future sewage load is 1681 p.e. This trunk line is not in danger of being overloaded by existing or future sewage loads.

Summary: No improvements necessary.

10. Area 28 into MH 28.

Area 28 is serviced by 8" service lateral or College Drive with a carrying capacity of 2970 p.e.

The existing sewage load in Area 28 is 5 p.e.
The future sewage load in Area 28 is 176 p.e.

Because Area 32 is also collected by this service lateral it has an existing sewage load of 18 p.e. and a future sewage load of 215 p.e. No improvements to this service lateral will be required to handle overloads.

EXHIBIT 6

1. MH 13 into MH 12.

The flow accumulated at MH 13 on Exhibit 5 includes the portion of Area 13 shown on this map. The 15" dia. main between MH 13 and MH 12 has a carrying capacity of 4755 p.e. The existing and future sewage loads through this line are 3459 p.e. and 5971 p.e., respectively, and this line will be overloaded by future development. A future 15" dia. parallel relief main is recommended as a future improvement.

Conclusion: Install 1317 ft. of 15" dia. relief main in the future.

2. Area 36 into MH 27,
MH 27 into MH 28.

Area 36 is serviced by an 8" service lateral on College Drive with a carrying capacity of 2235 p.e.

The existing sewage load in Area 36 is 249 p.e.
The future sewage load in Area 26 is 381 p.e.

This 8" service lateral will require no improvements to handle future sewage loads.

The 12" dia. trunk line between MH 27 and MH 28 has a carrying capacity of 2915 p.e. The existing flow through this line is 1534 p.e. and the future sewage load is 2062 p.e. This 12" dia. main line is adequate to prevent hydraulic overloading in the future.

Conclusion: No improvements necessary.

3. Area 32 into MH 28,
MH 28 into MH 29.

Area 32 is serviced by an 8" service lateral on College Drive with a carrying capacity of 2970 p.e.

The existing sewage load in Area 32 is 13 p.e.
The future sewage load in Area 32 is 39 p.e.

This service lateral also collects Area 28. The existing load in this service lateral is 18 p.e. and the future load is 215 p.e, and will require no improvements to handle overloads.

The 12" dia. trunk line between MH 28 and MH 29 has a carrying capacity of 2915 p.e. The existing flow through this line is 1552 p.e. and the future sewage load is 2277 p.e. This 12" dia. line is adequate to prevent overloading in the future.

Conclusion: No improvements necessary.

4. Area 33 into MH 29,
   Area 35 into MH 29,
   MH 29 into MH 30.

Area 33 flows into MH 29 through an 8" dia. service lateral with a carrying capacity of 1130 p.e.

The existing sewage load in Area 33 is 120 p.e.
The future sewage load in Area 33 is 120 p.e.

This 8" service lateral is sufficient.

Area 35 flows into MH 29 through an 8" dia. branch collector. Elevation data was not available to determine carrying capacities, but no problems are anticipated since both existing and future sewage load is only 167 p.e.

The 12" dia. trunk line between MH 29 and MH 30 has a carrying capacity of 2915 p.e. The existing flow through this line is 1839 p.e. and the future flow is 2564 p.e. This 12" dia. trunk line is sufficient to prevent overloading in the future.

Conclusion: No improvements necessary.

5. Area 40 into MH 30,
   MH 30 into MH 31.

Area 40 flows into MH 30 through an 8" dia. service lateral with a carrying capacity of 1585 p.e.

The existing sewage load in Area 40 is 97 p.e.
The future sewage load in Area 40 is 97 p.e.

This service lateral is sufficient to prevent overloading.

The 12" dia. trunk line between MH 30 and MH 31 has a carrying capacity of 2915 p.e. The existing sewage load is 1936 p.e. and the future sewage load is 2661 p.e. This line is sufficient to prevent hydraulic overloading in the future.

Conclusion: No improvements necessary.

6. MH 38 into MH 37,
   Area 43 into MH 37,
   Area 44 into MH 36,
   MH 37 into MH 36.

The 8" dia. service lateral between MH 38 and MH 37 has a carrying capacity of 980 p.e. The existing and future load through this line is 700 p.e., and no improvements are necessary.

Note: Area 43 cannot be serviced between MH 38 and MH 37 or this service lateral will become overloaded.

Area 43 is currently undeveloped but has a projected sewage load of 370 p.e.

Area 44 accumulates in the branch collector between MH 37 and MH 36.

The existing sewage load in Area 44 is 352 p.e.
The future sewage load in Area 44 is 417 p.e.

Note: 300 p.e. from the Town and County Mobile Home Park are serviced by an 8" dia. service lateral directly into MH 36.

The carrying capacity of the 8" dia. branch collector between MH 37 and MH 36 is 1205 p.e. The current sewage load is 752 p.e. and the future sewage load is 1187 p.e. No improvements to this branch collector are necessary to prevent future overloading.

Conclusion: No improvements necessary.

7. MH 36 into MH 34,
   Area 42 into MH 34.

The 8" dia. branch collector between MH 36 and MH 34 has a carrying capacity of 1105 p.e. The existing flow through this branch collector is 1052 p.e. and the future load is 1487 p.e. (the accumulated flows at MH 36). This branch collector will become overloaded in the future, especially with the development of Area 43. 950 feet of 8" dia. branch collection line is to be installed between MH 36 and MH 35 to relieve this line as future development warrants. This relief collector will reduce the future flow between MH 36 and MH 34 to 744 p.e.

Area 42 is currently undeveloped but has a future projected sewage load of 343 p.e.

When Area 42 and Area 43 are developed and the 950 ft. - 8" dia. relief main is built, the accumulated flow at MH 34 will be 1087 p.e. The existing accumulated flow at MH 34 is 1052 p.e.

Note: Area 42 cannot be developed without the relief sewer from MH 36 to MH 35!

8. Area 48 into MH 33,
   Area 41 into MH 33,
   MH 34 into MH 33.

The existing sewage load in Area 48 is 39 p.e.
The future sewage load in Area 48 is 39 p.e.

The carrying capacity of the 8" dia. branch collector between MH 34 and MH 33 is 1205 p.e. The existing flow through this line is 1091 p.e. and the future flow with the recommended improvement upstream is 1126 p.e. Only 79 p.e. (30 residential lots) are available for service to Area 41 by this line.

Area 41 is serviced by an 8" dia. branch collector on Allison Road with a carrying capacity of 1315 p.e.

The existing sewage load in Area 41 is 80 p.e.
The future sewage load in Area 41 is 344 p.e.

No improvements to this branch collector will be necessary to prevent overloading. Development in Area 41 needs to be routed mainly to Allison Road as the branch collector between MH 34 and MH 33 can only receive flow from 30 residences in Area 41. The accumulated flow at MH 33 will be 1171 for the existing load and 1470 p.e. for the future sewage load.

Conclusion: No improvements are necessary.

9. MH 33 into MH 31,
   Area 39 into MH 31.

The 8" dia. branch collector between MH 33 and MH 31 has a carrying capacity of 1105 p.e. The existing flow through this line is 1171 p.e. and flows at 62% full under maximum loading conditions (peak flow with Holiday Inn, fully occupied). The future sewage load is 1470 p.e. In addition, Area 39 accumulates in the collection line between MH 33 and MH 31. Area 39 is currently undeveloped, but the future population is projected at 343 p.e. Therefore, 1813 p.e. will be serviced by the branch collector between MH 33 and MH 31. A parallel 8" dia. relief branch collector will need to be installed between MH 33 and MH 31 as future development necessitates.

10. MH 31 and MH 32.

Flow from MH 33 to MH 31 and flow from MH 30 to MH 31 combine to become the flow from MH 31 to MH 32. The existing flow from MH 31 to MH 32 is 3107 p.e. The future flow from MH 31 to MH 32 is 3107 p.e. The future flow from MH 31 to MH 32 is 4475 p.e.

The 12" dia. trunk line from MH 31 into MH 32 has a carrying capacity of 3055 p.e. and currently flows at 61% full during peak loading conditions. 375 ft. of 12" dia. relief main is recommended as an immediate improvement.

Conclusion: Install 375 ft. of 12" dia. relief main parallel to the existing 12" dia. sanitary main now.

11. Area 34 into MH 32,
    Area 38 into MH 32,
    MH 32 into MH 17.

Area 38, currently undeveloped, will flow into the north invert of MH 32. Area 34 collects in an 8" dia. service lateral with a carrying capacity of 2495 p.e. and flows into the south invert of MH 32. These join the flow from MH 31 into MH 32 above, and continues on to MH 17.

The existing sewage load from Area 34 is 121 p.e.
The future sewage load from Area 34 is 121 p.e.
The existing sewage load from Area 38 is 0 p.e.
The future sewage load from Area 38 is 53 p.e.

The total existing sewage load from MH 32 to MH 17 is 3228 p.e. and the future sewage load through this collection main is 4649 p.e. The carrying capacity of this trunk line is 3055 p.e, and currently flows 62% full during peak loading conditions. A 12" dia. relief main is needed now, and will raise the carrying capacity between MH 32 and MH 31 to 6055 p.e.

Conclusion: Install 375 ft. of 12" dia. relief main now.

12. MH 36 into MH 35,
Area 37 into MH 17,
MH 35 into MH 17.

The 950 ft. - 8" dia. relief branch collector from MH 36 to MH 35 previously mentioned, (to relieve the 8" dia. branch collector to the west), will contribute at MH 35 a future sewage load of 743 p.e. (half of the sewage load from Areas 43, 44 and 45). Area 37 will accumulate with this inflow to its point of discharge in the 12" dia. trunk line at MH 17.

The existing sewage load from Area 37 is 155 p.e.
The future sewage load from Area 37 is 207 p.e.

This results in loading the 8" dia. collection line between MH 36 and MH 17 with an existing load of 155 p.e. and future load of 950 p.e. This 8" lateral has a carrying capacity of 980 p.e. and will be sufficient to handle future loads.

Note: This 8" dia. line between MH 35 and MH 17 has a slope of 0.003 ft./ft. (too flat by current design standards). To a point, increased flow will improve this line by increasing the velocity of flow. However, ventilation becomes critical at greater flows so the construction of the relief sewer between MH 36 and MH 35 must place the inverts at the same elevation and slopes out of MH 36 must also be equal. Survey data control is essential.

The total accumulation at MH 17 is this flow between MH 35 and MH 17, combined with the flow from MH 32 on Prosser Road. The cumulative flow at MH 17 is 3383 p.e. for the existing sewage load and 5599 p.e. for the future sewage load.

13. MH 17 into MH 12,
Area 12 into MH 12.

Area 12 accumulates in the 12" dia. trunk line on Prosser Road between MH 17 and MH 12. Several 8" dia. service laterals and branch collectors are the source of this load.

A road bore is included in the sewer between MH 17 and MH 12. The carrying capacity of the 12" dia. main line through the road bore is 3055 p.e. and its existing sewage load is 3383 p.e. The future sewage load will be 5599 p.e. A parallel 12" dia. relief main under the highway is needed now.

The existing sewage load from Area 12 is 1093 p.e.
The future sewage load from Area 12 is 1158 p.e.

The remaining main line between MH 17 and MH12 has a carrying capacity of 3255 p.e. The existing sewage load is 4476 p.e. The future sewage load will be 6757 p.e. A parallel 12" dia. relief main is needed here now.

Conclusion: Install 2750 ft. of 12" dia. relief main, including 108 ft. under Highway 85, now.

14. Area 11 into MH 11.

An 8" dia. branch collector on Allison Road services area 11. This branch collector has a carrying capacity of 1305 p.e.

The existing sewage load in Area 11 is 229 p.e.
The future sewage load in Area 11 is 678 p.e.

No improvements in the collection system in this area will be required to prevent hydraulic overloading.

15. Area 4 into MH 10,
Area 5 into MH 10,
MH 10 into MH 9,
Area 7 into MH 9.

Areas 4,5, and 7 are serviced by an 8" dia. branch collector on Jefferson Road. The carrying capacity of this branch collector between MH 10 and MH 9 is 1140 p.e.

The existing sewage load from Area 4 is 232 p.e.
The future sewage load from Area 4 is 442 p.e.
The existing sewage load from Area 5 is 0 p.e.
The future sewage load from Area 5 is 132 p.e.
The existing sewage load from Area 7 is 152 p.e.
The future sewage load from Area 7 is 152 p.e.

This results in an existing sewage load from MH 10 to MH 9 of 384 p.e. and a future sewage load of 726 p.e. No improvements will be necessary to prevent hydraulic overloads of this line.

16. MH 39 into MH 9,
Area 6 into MH 9,
MH 9 into MH 8.

The existing 8" dia. branch collector between MH 39 and MH 9 has a carrying capacity of 1495 p.e. Area 6 accumulates with the inflow to MH 39 (from Exhibit 2) between MH 39 and MH 9.

The existing sewage load from Area 6 is 45 p.e.
The future sewage load from Area 6 is 177 p.e.

The total sewage flow through this line is 233 p.e. for existing and 365 p.e. for future propulations. No improvements will be necessary in this branch collector.

The accumulated flow at MH 9 is 617 p.e. for existing and 1091 p.e. for future loads. The 8" dia. branch collector between MH 9 and MH 8 has a carrying capacity of 1130 p.e. and will be sufficient to handle these flows.

17. Area 8 into MH 8,
MH 8 into MH 7.

The flow from MH 9 into MH 8 (above) combines with the flow from Area 8 into MH 8 and flows into MH 7. Area 8 is serviced by an 8" dia. service lateral on Lake Place with a carrying capacity of 1130 p.e.

The existing sewage load from Area 8 is 27 p.e.
The future sewage load from Area 8 is 291 p.e.

The existing flow from MH 8 to MH 7 is 644 p.e. and the future flow will be 1382 p.e. This 8" dia. branch collector has a carrying capacity of 1130 p.e. Future flows will overload this line and replacement with 10" dia. pipe is recommended.

Conclusion: Replace 300' - 8" dia. with 10" dia. pipe as future development necessitates.

18. Area 9 into MH 7,
MH 7 into MH 6.

The flow from MH 8 into MH 7 (above) combines with the flow from Area 9 at MH 7 and flows into MH 6. Area 9 is serviced by an 8" dia. service lateral on Tyler Place.

The existing sewage load from Area 9 is 147 p.e.
The future sewage load from Area 9 is 147 p.e.

The existing flow from MH 7 to MH 6 is 791 p.e. and the future flow is 1529 p.e. The carrying capacity of this 8" dia. branch collector is 1130 p.e. Future flows will overload this line and replacement with 10" dia. pipe is recommended when future development necessitates.

Conclusion: Replace 915 ft. of 8" dia. pipe with 10" dia. pipe in the future.

19. Area 10 into MH 6,
MH 6 into MH 5.

The flow from MH 7 to MH 6 (above) combines with the flow from Area 10 at MH 6 and continues on to MH 5. The 8" dia. branch collector between MH 6 and MH 5 has a carrying capacity of 1130 p.e., and the 8" dia. service lateral on Gopp Court servicing Area 10 has a carrying capacity of 1350 p.e.

The existing sewage load from Area 10 is 58 p.e.
The future sewage load from Area 10 is 190 p.e.

The total flow from MH 6 to MH 5 is 849 p.e. for existing and 1719 p.e. for future flows. This 8" ida. branch collector will become overloaded in the future, at which time replacement with 10" dia. pipe is recommended.

Conclusion: Replace 210' - 8" dia. with 10" dia. pipe in the future.

EXHIBIT 7

1. Area 19 into MH 12,
   MH 12 into MH 3,
   MH 12 into MH 11.

The flow from MH 13 into MH 12 and the flow from MH 17 to MH 12 (on Exhibit 6) combine with the flow from Area 19 into MH 12, resulting in the total inflow to MH 12.

The existing sewage load from Area 19 is 37 p.e.
The future sewage load from Area 19 is 37 p.e.

The total inflow to MH 12 is 7972 p.e. for existing flow and the total inflow to MH 12 is 12,764 p.e. for future flow.

The 18" dia. trunk line between MH 12 and MH 11 has a carrying capacity of 7735 p.e. and is currently the only source of outflow from MH 12. It currently flows 62% full under peak loading conditions. An 18" dia. cross interceptor from MH 12 to MH 3 is recommended for relief of this sewer main. This 18" dia. cross interceptor relief main should be constructed in the following manner to provide proper relief in the existing main trunk line downstream. (MH 11 to MH 5 to MH 4 to MH 3):

1. Provide a slope of 0.0037 ft./ft. uniformly between MH 12 and MH 3 on the new relief sewer. This will provide a carrying capacity of 9450 p.e. in the new sewer line at 60% full.

2. Place the invert of this relief sewer 0.23 ft. below the invert elevation of the existing outflow line (from MH 12 to MH 11). This will force the relief sewer to carry 8,764 p.e. at 57% of the full depth and the existing main to carry 4,000 p.e. at 42% of the full depth when the projected future flow occurs.

The existing flow from MH 12 to MH 11 is 7,972 p.e. and the future flow will be 4,000 p.e., with this recommended improvement.

Conclusion: Install 2,000 ft. of 18" dia. relief main now.

2. Area 18 into MH 11,
   MH 11 into MH 5.

Area 18 is serviced by an 8" dia. branch collector and flows into the 18" dia. trunk line between MH 12 and MH 11.

The existing sewage load from Area 18 is 87 p.e.
The future sewage load from Area 18 is 87 p.e.

The flow from MH 12 combines with this flow and that from Area 11 into MH 11 (from Exhibit 6) and continues on in the 18" dia. sewer main with a carrying capacity of 7,735 p.e. from MH 11 to MH 5.

The existing flow from MH 11 to MH 5 is 8,288 p.e. and the future flow will be 4,765 p.e., with the improvements recommended above.

Note: Without the cross interceptor from MH 12 to MH 3 the future sewage load would be 13,529 p.e.

Conclusion: The main line from MH 11 to MH 5 is overloaded at this time, but will not be overloaded in the future.

3. Area 17 into MH 5,
   MH 5 into MH 4.

Area 17 flows into MH 5 and combines with the flow from MH 11 to MH 5 (above) and the flow from MH 6 to MH 5 (Exhibit 6) and this cumulative flow is carried from MH 5 to MH 4.

The existing sewage load from Area 17 is 138 p.e.
The future sewage load from Area 17 is 138 p.e.

The existing sewage load from MH 5 to MH 4 is 9,275 p.e. and the future sewage load is 6,622 p.e. The 18" dia. main line from MH 5 to MH 4 has a carrying capacity of 8,830 p.e., is currently overloaded, but will not be in the future.

Note: Without the improvement from MH 12 to MH 3, the future sewage load through this line would be 15,386 p.e.

4. Area 2 into MH 4,
   MH 4 into MH 3.

The flow from MH 5 to MH 4 (above) combines with the flow from Area 2 and it flows in the 18" dia. main with a carrying capacity of 7,580 p.e. from MH 4 to MH 3. Area 2 is serviced by an 8" dia. service lateral with a carrying capacity of 1,435 p.e.

The existing sewage load from Area 2 is 155 p.e.
The future sewage load from Area 2 is 287 p.e.

The existing flow from MH 4 to MH 3 is 9,430 p.e. and the future flow will be 6,909 p.e. This line is currently overloaded and the relief sewer from MH 12 to MH 3 is needed now.

Note: Without the recommended improvement the future load from MH 4 to MH 3 would be 15,673 p.e., which is the combined flow at MH 3 in either case.

5. MH 3 into MH 2.

The 18" dia. main line between MH 3 and MH 2 has a carrying capacity of 7,580 p.e. The existing sewage load between MH 3 and MH 2 is 9,430 p.e. and the future sewage load will be 15,673 p.e. A parallel 18" dia. relief sewer is recommended to be constructed now. When the future population of 15,000 residents is reached, these parallel lines will be operating at 62% full.

Conclusion: Install 2,400 ft. of 18" dia. line now.

6. Area 20 into MH 2,
MH 2 into MH 1.

Area 20 is serviced by a private line from Laramie County Community College. The existing and future sewage load from LCCC is estimated at 100 p.e. The existing 18" dia. main line from MH 2 to MH 1 has a carrying capacity of 7,580 p.e. The existing sewage flow through this line is 9,530 p.e. and the future sewage load will be 15,773 p.e. A parallel 18" dia. relief main is recommended to be installed parallel to the existing main. The private line from LCCC should be intercepted by this relief sewer.

Conclusion: Install 1,950 ft. of 18" dia. relief sewer now.

7. MH 40 into MH 1.

The 8" dia. branch collector between MH 40 and MH 1 has a carrying capacity of 1,145 p.e. The existing sewage load through this line is 750 p.e. and the future sewage load will be 1,173 p.e.

This branch collector will flow at 63% of the full depth with future flows. No improvements are recommended for this line, but future proposed development (from Area 1) will load this line to its usage limit.

8. MH 1 to STP.

The accumulated flows at MH 1 are 10,280 p.e. for existing flow and 16,946 p.e. for future flow. The 21" dia. main from MH 1 to the sewage treatment plant has a carrying capacity of 8,450 p.e. and is currently flowing 68% full. A parallel 21" dia. relief main is recommended to be installed now. This will increase the carrying capacity from MH 1 to the STP to 16 900 p.e., which is sufficient to handle the flows for the projected design population of 15,000 residents.

Conclusion: Install 3,980 ft. of 21" dia. relief main now.

SECTION 5

RECORDING FUTURE DEVELOPMENT AND PROJECTING OVERLOADS

Tables 1,2,3, and 4 are provided to enable the District to keep records of future development approvals and to decide when improvements to the existing system should be made. Updating these tables has the effect of tracing increases in flow from any area all the way to the sewage treatment plant.

TABLE 1 - Lists each area's existing and future population. As each area is developed, the existing approved population should be updated. If requests are made for development beyond the estimated future population, these requests should be studied to determine what influence these developments will have on the system.

TABLE 2 - Lists each manhole identified in this study, lists the existing and future flows into them, and the areas which contribute flow to that particular manhole. These records should also be updated in the same manner as Table 1 when any new development is approved.

TABLE 3 - Lists the sewer lines that need immediate relief, with their carrying capacities and their current loads.

TABLE 4 - Lists the sewer lines that will need relief in the future with their carrying capacity, existing flow, available increase, and contributory areas. This table should be updated for each new approved development.

These tables should be updated on the basis of the following chart:

POPULATION EQUIVALENT CONVERSION CHART (p.e.)**

Residential Lot	2.64 p.e.
Trailer Space	3.00 p.e.
Commerical Development	13.00 p.e.
Apartment Building, per Single Family Dwelling	2.50 p.e.
Industry, Institution	*

* As determined by an estimate of water usage. Use average use in gallons per day divided by 100 for the number of population equivalents.

** Please refer to Page 10 of this report for a definition and example of population equivalents (p.e.).

SUMMARY TABLE 1

Area #	Existing Approved Population	Estimated Future Population
A1	750	1,173
A2	155	287
A3	188	188
A4	232	442
A5	0	132
A6	45	177
A7	152	152
A8	27	291
A9	147	147
A10	58	190
A11	229	678
A12	1,093	1,158
A13	160	318
A14	285	461
A15	161	363
A16	161	424
A17	138	138
A18	87	87
A19	37	37
A20	100	100
A21	618	768
A22	411	1,385
A23	185	434
A24	85	176
A25	261	378
A26	300	300
A27	0	132
A28	5	176
A29	232	232
A30	156	156
A31	597	993
A32	13	39
A33	120	120
A34	121	121
A35	167	167
A36	249	381
A37	155	207
A38	0	53
A39	0	343
A40	97	97
A41	80	344
A42	0	343
A43	0	370
A44	352	417
A45	700	700
A46	1,048	1,048
A47	84	84
A48	39	39
A49	0	0
Total	10,280	16,946

SUMMARY TABLE 2.

MH #	Existing Flow Into	Future Flow Into	Contributory Areas
1	10,280	16,946	A1-A49
2	9,530	15,773	A2-A49
3	9,430	15,673	A2-A19, A21-A49
4	9,430	6,909	A2-A19,A21-A49
5	9,275	6,622	A3-A19, A21-A49
6	849	1,719	A2-A10
7	791	1,529	A3-A9
8	644	1,382	A3-A8
9	617	1,091	A4-A7
10	232	574	A4, A5
11	8,288	4,765	A11-A16,A18,A19,A21-A49
12	7,972	12,764	A12-A16,A19,A21-A49
13	3,459	5,971	A13-A16,A21-A25,A27,A46,A47
14	3,299	5,653	A14-A16,A21-A25,A27,A46,A47
15	3,014	5,192	A15,A16,A21-A25,A27,A46,A47
16	2,592	4,319	A16,A21-A24,A46,A47
17	3,383	5,599	A26,A28-A45,A48
18	261	510	A25,A27
19	2,431	3,895	A21-A24,A46,A47
20	618	768	A21,A49
21	411	1,385	A22,A49
22	1,299	2,763	A21-A24
23	2,431	3,895	A21-A24,A46,A47
24	1,048	1,048	A46
25	597	993	A31
26	1,053	1,449	A26,A30,A31
27	1,534	2,062	A26,A29-A31,A36
28	1,552	2,277	A26,A28-A32,A36
29	1,839	2,564	A26,A28-A33,A35,A36
30	1,936	2,661	A26,A28-A33,A35,A36,A40
31	3,107	4,475	A26,A28-A33,A35,A36,A39-A45,A48
32	3,228	4,649	A26,A28-A36,A38-A45,A48
33	1,171	1,470	A41-A45,A48
34	1,052	1.087	A42-A45
35	0	743	A43-A45
36	1,052	1,487	A43-A45
37	700	1,070	A43,A45
38	700	700	A45
39	188	188	A3
40	753	1,173	A1

Summary Table 3

Lines for Immediate Improvement	Carrying Capacity p.e.	Existing Flow p.e.
1. MH1 to STP	8540	10,280
2. MH2 to MH1	7580	9,530
3. MH3 to MH2	7580	9,430
4. MH12 to MH3	*	7,972
5. MH17 to MH12	3255	4,476
6. MH32 to MH17	3055	3,228
7. MH31 to MH32	3055	3,383

* Carrying Capacity in existing lines vary; all lines currently overloaded

Summary Table 4

Lines for future Improvement	Carrying Capacity	Existing Flow p.e.	Available Increase p.e.	Contributory Areas
1. MH33 to MH31	1105	1171	none	A39, A41-A45, A48
2. MH36 to MH34	1105	1052	53	A43-A45**
3. MH8 to MH7	1130	644	486	A3-A8
4. MH7 to MH6	1130	791	339	A3-A9
5. MH6 to MH5	1130	849	281	A3-A10
6. MH25 to MH26	980	597	383	A31
7. MH13 to MH12	4755	3459	1296	A13-A16, A21-A24, A46, A47, A49
8. MH14 to MH13	4325	3299	1026	A14-A16, A21-A24, A46, A47, A49
9. MH15 to MH14	3940	3014	926	A15, A16, A21-A24, A46, A47, A49

Summary Table 4 Cont.

Lines for future Improvement	Carrying Capacity	Exisitng Flow p.e.	Available Increase p.e.	Contributory Areas
10. MH16 tc MH15	3940	2597	1348	A16, A21-A24, A46, A47, A49
11. MH19 to MH16	2635	2592	43	A21-A24, A46, A47, A49
12. MH23 to MH19	2635	2431	204	A21-A24, A46, A47, A49

** When areas A43, A44, and A45 develop by 53 p.e., construct relief sewer from MH36 to MH35.

APPENDIX A

INDEX TO III

SECTION 4

DETAILED ANALYSIS BY AREA AND MANHOLE

PART B

SOUTH CHEYENNE WATER AND SEWER DISTRICT

PRELIMINARY DRAINAGE EVALUATION

INTRODUCTION

AVI p.c. has been contracted by the Office Of The Industrial Siting Administration to provide a Preliminary Master Drainage Plan for the South Side Water and Sewer District, Cheyenne, Wyoming. The South Side Water and Sewer District is located within Sections 8,17,20; and a portion of Section 5 south of Interstate 80 bordered by Person Road, Ave. B-, East Fox Farm Road and South House; a portion of Section 4 South of Interstate 80 bordered by Gordon Road, Avenue D, Avenue C-4; a portion of Section 9, east of Avenue C to the old railroad grade, south of East Fox Farm Road, north of East Jefferson Road; Township 13 North, Range 66 West, 6th P.M. Laramie County, Wyoming.

The objectives of this study are:

1. Develop a Preliminary Master Drainage Plan.

2. Address Method and Areas

3. Address Problem Areas

4. Recommendations.

PRELIMINARY MASTER PLAN

A Preliminary Master Plan was developed to provide an opportunity for unified drainage. It should be maintained in an up to date fashion at all times for each drainage basin and sub-basin to reflect changes due to urbanization and modified natural water courses. This plan initially covers only the major drainage facilities to ensure identification of discharge and outfall points, while at the same time, giving enough detail to provide a ready drainage development guide for the future in any particular sub-basin of the District. Final hydrological analysis should be performed for additional refinements and use. The completed Preliminary Master Plan is suitable for day to day use to determine effects of probable future ultimate development within the District as it effects both hydrology and hydraulic design.

RATIONAL METHOD

The South Side Water and Sewer District is divided into sub-basins by U.S. Highway 85 in the north - south direction and College Drive in the east-west direction. Surface water run-off west of U.S. 85 is directed north along natural watercourses to an existing 42" R.C.P. culvert at the intersection of U.S. 85 with Artesian Road, and two 4' x 6' box culverts which transport water under U.S. 85 between Allison and Prosser Roads. Surface water south of College Drive and East of U.S. 85 is directed northeast to the Crow Creek.

Surface water north of College Drive and east of U.S. 85 is directed eastward to the Crow Creek. The City of Cheyenne and the County of Laramie require that only the historic (pre-development) flows be allowed to pass. Therefore an assessment of flood flows from areas of proposed future development, before and after development must be made.

The Rational Formula shall be the method used in the Cheyenne Region to compute the amount of peak flows. The Rational Formula is:

$$Q = C\ I\ A\ C_f$$

Where:

Q = peak discharge (cfs)

C = run-off coefficient

I = rainfall intensity (in/hr)

A = area (acres)

C_f = correction factor for design storm

The peak discharges determined by this method are approximate. Rarely will the drainage facility operate at the design discharge. Flow will always be more or less in actual practice merely passing the design flow as it rises and falls. The following watershed data are required to estimate the pre and post development peak flows.

1. Map showing topography, streams and run-off contributing drainage and;
2. Information about soils and vegetative cover, development, and their distribution throughout the watershed.

Specific input into the Rational Method includes drainage area (acres), length of the longest watercourse (feet), elevation difference of longest watercourse (feet), run-off coefficient, and rainfall intensity vs. duration distribution curves for the different frequency storms.

The length of the longest watercourse and the elevation differences of the longest watercourse were used along with the velocity information to give a preliminary estimate of the time of concentration.

Precipitation values associated with the 100 yr. and 5 yr. return periods are applied to each drainage basin to calculate surface, run-off. Rainfall data presented in this report was taken from data for the Cheyenne Area. (NOAA, 1973). The intensity-duration-frequency graphs for the Cheyenne Area were taken from the Drainage Management Manual (draft).

The run-off coefficient C, is determined using a weighted average of various surface characteristics. An area that is 25% asphalt (C=0.95), 60% gentle slope native grass (C=0.15) and 15% residential (1/2 acre lots or more; C=0.35) has an average coefficient of [(.25)(.95) + (.60)(.15) + (.15)(.35)] = 0.38.

Peak flow estimates for the various basins and sub-basins need to be combined to determine the pre and post development inflows to the outfall points. It is assumed that areas upstream of the existing culverts under U.S. Highway 85 will detain the volume of water in excess of the capacity of the culverts. This assumption is reasonable based upon field reconnaissance of those existing culverts.

An outflow hydrograph from the culverts could not be generated without specific topographic details describing the already developed areas upstream of the culverts. Therefore, an extensive routing analysis, using the Manning equation to find the velocity of flow, was not done. The use of the Rational Method with an extensive routing analysis may result in a more feasible drainage design. The peak flow obtained by summing the peak flows from individual drainage basins provides a conservative estimate of the peak flow reaching the outfall points.

The pre-development 100 year run-off estimates for each sub-basin are presented in Appendix B. The post-development peak run-off, as proposed future development takes place, should not exceed the pre-development peak flows. The major drainage characteristics which may change due to future development are drainage area, length of longest watercourse, and coefficient of run-off. The effect of development (streets and gutters, buildings, etc.) is accounted for in the weighted average coefficient of run-off. The excess run-off volume created by future development must be stored in some form of detention facility and discharged at a rate equivalent to or less than the 5 year pre-development peak flow. Again, due to lack of detailed information, the peaks from each basin were arithmetically summed to provide a conservative estimate.

PROBLEMS WITH EXISTING DRAINAGE

The major drainageways in the District are well defined and the initial drainage system is designed. The initial drainage system transporting storm run-off consists of property line swales, streets and gutters, storm sewers, roadside drainage ditches, and culverts designed to handle run-off from the initial storm. Planning and design of existing storm water drainage systems look to be based on the premise that problems could be transferred from one location to another. Urban development is not located wisely in respect to the major drainage system. Drainage planning must have been done after all other decisions were already made as to the layout of new subdivisions. Some developments never received full site planning and engineering analysis to adequately handle the drainage. There are certain areas built within the district which will not conform to drainage standards required by the City of Cheyenne and the County of Laramie.

The upgrading of these areas to conform to all policy, criteria, and standards in the Drainage Management Manual will be difficult, if not impractical to obtain short of complete redevelopment or renewal.

Drainageways have been obliterated by development such as in the area of Allison Tracts 53, 54, 55. No provision to re-establish this natural water course with an open channel to convey storm run-off water was made. Streets have been used as floodways for initial storm run-off. Such is the case with Williams Street in Galaxy Estates.

The initial system no longer has the capacity for additional run-off. Storm water run-off occurs no matter how poorly the planning and design to control storm water run-off is done. The quality of the planning determines the cost to the developer, to the community, and effects on the residents and future urban developments.

Future development of the South Side Water and Sewer District is inevitable. The ultimate development anticipated is the addition of 70 commercial establishments and a population increase of 6,600 persons. Subdivisions for residential purposes will be mostly single family housing and mobile home parks.

Areas of proposed urban growth are at this time undeveloped. The future types of development within these undeveloped basins will substantially increase the amount of storm water run-off. Adequate provisions for the drainage demand must be met as changes due to urbanization effect both the hydrology and hyraulic design of the basin.

RECOMMENDATIONS

The Preliminary Master Plan developed for storm drainage should be regularly reviewed and updated to reflect changes due to urbanization and changed channel conditions. Future development efforts should coordinate with the predetermined objectives of the Master Plan. Planning and design of storm water run-off facilities should not be based on the premise that problems can be transferred from one location to another. If adequate provision is not made in the land use plan for the drainage demand, storm water run-off will conflict with other land uses. Drainage planning should not be done after all other decisions are already made as to layout of a new subdivision or commercial area. It is this latter approach which creates problems which are costly to correct latter down the line.

Before commencing design of any drainage facilities, comprehensive facts and data should be collected and examined for the particular site location. The upgrading effect of development on the rate of flow should address downstream effects in each sub-basin. All land development proposals should receive full site planning and engineering evaluation.

The planning of urban drainage should proceed with a defined set of drainage policies backed by suitable subdivision regulations. We strongly recommend that all land development strictly adhere to subdivision regulations to assure proper construction and successful operation and maintenance of drainage facilities. Drainage design should have as an objective the reduction of street repair and maintenance costs to the public.

Good land planning should reflect even minimal thalwegs and natural watercourses to reduce development cost and minimize drainage problems. The wise utilization of natural watercourses and other drainage channels running through the site may eliminate the need for costly underground storm sewers. Encroachment upon or land modifications within these natural watercourses should not be permitted under most circumstances. It must be remembered that the major system exists in a community whether or not urban development is situated wisely with respect to it. Water will seek its lowest level despite developments in its way. Where drainage ways have been obliterated by development these must be re-established. They are essential features of the major drainage system.

In the initial drainage system, storm water run-off is transported via property line swales, drainage ditches , culverts, streets and gutters, storm sewers, and other features designed to handle run-off from the 5 year storm. The use of streets shall fully recognize that the primary function for streets is traffic movement. Streets should not be used as floodways for initial storm run-off. The City or County officials should think in terms of natural drainage easements and street drainage patterns. Good planning of streets can substantially help in eliminating the need for a storm sewer system. It is perhaps at this point of the planning process where the greatest impact can be made as to what drainage facilities will cost. The earlier the drainage problems are identified and planned for, the lower the cost of drainage facilities.

Drainage facilities can fulfill a number of purposes. In addition, drainage facilities not necessarily designed primarily for drainage may be incorporated to provide drainage benefits. Storage of storm run-off close to the point of rainfall occurance include use of roof-tops, parking lots, property line swales, road embankments, barrow pits and on site detention ponds. By utilizing these facilities for storage of storm water run-off, the drainage capacity downstream is reduced, thus reducing the land area and substantially reducing the costs of the system and its maintenance.

For drainage purposes, the effects of maintenance or lack of it becomes very important in the operation of drainage facilities. Proper maintenance is often ignored resulting in a deterioration of even a well designed system. Roadside drainage ditches deteriorate due to scour from high velocity, sedimentation at low velocity or ponded water. Detention ponds fill up with debris and sedimentation and culverts become obstructed and eventually erode. Without maintenance, these facilities become unsightly, a social liability and eventually ineffective in handling storm water run-off. Again, it must not be ignored if the drainage system is to remain operational and benefit the community.

Drainage is a priority. Planning for drainage should precede all other development. The following items are necessary to provide quality drainage planning to better benefit the community.

1. Maintain Preliminary Master Plan up to date.
2. Require full site planning and engineering.

3. Strictly adhere to subdivision regulations regarding drainage management.
4. Require proper maintenance to keep drainage facilities operational.

Returns to the community can be great when the planning precedes development.

APPENDIX B

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
1.	38.4	2075	2.6		13.8	2.45	0.29	34.1
2.	8.6	700	2.2		10.6	2.45	0.2	5.3
3.	4.38	1400	3.4		16.7	2.45	0.2	2.7
4.	195.5	5600	1.7		109.8	0.48	0.2	23.5
5.	236.0	5500	1.2		114.6	0.48	0.2	28.32
6.	9.1	950	0.5		14.4	2.45	0.27	7.5
7.	26.8	2000	2.9		13.3	2.45	0.25	20.5
8.	71.9	2500	1.2		23.8	2.45	0.34	74.9
9.	4.2							-
10.	6.1	1350	1.5		12.9	2.45	0.2	3.7
11.	10.8	1000	2.3		7.4	2.45	0.2	6.6
12.	4.0	750	1.2		8.3	2.45	0.2	2.5
13.	5.0	650	1.2		9.2	2.45	0.2	3.1
14.	3.5	750	.75		9.6	2.45	0.2	2.1
15.	4.4	700	3.3		4.2	2.45	0.2	2.7
16.	6.0	850	1.2		12.9	2.45	0.39	7.2
17.	14.3							-

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

AREA acres		L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
18.	159.8	5000	3.4		27.8	2.45	0.2	97.9
19.	35.6	3400	3.9		18.9	2.45	0.22	24.0
20.	20.2	1750	3.3		10.6	2.45	0.30	18.6
21.	20.8	2600	5.0		13.3	2.45	0.30	19.1
22.	2.9	400	1.0		4.4	2.45	0.34	3.0
23.	0.6	200	3.0		1.3	2.45	0.25	0.46
24.	1.6	450	1.3		4.3	2.45	0.34	1.7
25.	6.2	850	1.0		9.4	2.45	0.40	7.6
26.	5.3	600	1.8		4.5	2.45	0.32	5.2
27.	1.3	425	1.7		3.5	2.45	0.20	0.8
28.	3.1	725	1.5		6.9	2.45	0.34	3.2
29.	11.1	950	1.4		9.1	2.45	0.34	11.6
30.	6.3	1500	0.5		20.0	2.45	0.40	7.7
31.	2.1	600	1.0		6.7	2.45	0.34	11.6
32.	10.7	850	1.2		8.3	2.45	0.40	13.1
33.	2.5	475	1.7		4.0	2.45	0.40	3.1
34.	2.7	700	1.3		6.7	2.45	0.34	2.8
35.	8.5	1000	1.1		10.4	2.45	0.40	10.4
36.	5.4	700	1.0		7.3	2.45	0.40	6.6

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
37.	10.4	1750	1.3		16.7	2.45	0.40	12.7
38.	127.2	3600	0.5		54.5	0.48	0.20	15.26
39.	9.5	800	0.63		10.7	2.45	0.40	11.6
40.	1.2	400	2.5		2.6	2.45	0.34	1.2
41.	6.9	750	1.5		7.1	2.45	0.40	8.5
42.	26.4	2400	2.0		18.6	2.45	0.20	16.2
43.	15.4	1000	0.5		15.2	2.45	0.40	18.9
44.	5.8	700	0.9		10.0	2.45	0.34	6.0
45.	12.1	1300	1.9		9.6	2.45	0.34	12.6
46.	26.5	2500	1.2		27.8	2.45	0.24	19.33
47.	6.1	1000	1.25		9.5	2.45	0.20	3.7
48.	3.6	800	1.1		8.9	2.45	0.20	2.2
49.	20.3	1500	1.9		11.4	2.45	0.20	12.4
50.	18.3	1400	1.5		13.3	2.45	0.20	11.2
51.	24.6	2200	5.3		11.1	2.45	0.30	22.6

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
52.	120.2	7400	1.2		82.2	0.48	0.25	18.0
53.	86.7	3700	3.4		22.4	2.45	0.20	53.1
54.	38.6	2600	4.5		14.0	2.45	0.20	23.6
55.	31.0	2000	5.75		10.25	2.45	0.20	19.0
56.	20.8	1500	6.0		7.6	2.45	0.20	12.7
57.	44.5	2100	0.7		28.0	2.45	0.36	49.06
58.	7.1	675	1.4		6.6	2.45	0.30	6.5
59.	2.7	650	1.2		6.8	2.45	0.36	3.0
60.	43.4	2500	1.5		23.1	2.45	0.30	39.9
61.	13.3	1800	2.8		12.0	2.45	0.30	12.2
62.	24.9	1500	1.5		13.9	2.45	0.30	22.9
63.	7.2	1300	1.0		14.4	2.45	0.22	4.9
64.	22.3	1500	1.0		16.7	2.45	0.30	20.5
65.	5.4	750	2.8		5.0	2.45	0.20	3.3
66.	4.1	600	2.1		4.4	2.45	0.38	4.8
67.	47.0	2800	3.0		7.3	2.45	0.21	30.2
68.	4.9	850	7.0		4.0	2.45	0.20	3.0

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
69.	59.6	4000	2.5		26.7	2.45	0.36	65.7
70.	5.4	400	5.0		2.1	2.45	0.20	3.3
71.	18.4	1950	1.7		16.3	2.45	0.56	31.6
72.	125.8	6800	1.0		161.9	0.48	0.22	66.6
73.	221.4	5900	2.7		81.9	0.48	0.20	26.6
74.	216.0	7400	3.0		98.7	0.48	0.20	25.9
75.	38.3	3300	2.0		55.	0.48	0.20	4.6
76.	259.7	10300	2.3		156.	0.48	0.20	31.2
77.	60.5	3850	1.0		42.8	0.48	0.32	11.6
78.	21.8	2400	0.6		32.0	0.48	0.23	3.0
79.	35.5	1500	1.25		14.3	2.45	0.27	29.4
80.	21.0	1400	3.3		8.5	2.45	0.24	15.4
81.	55.9	2500	0.5		37.9	2.45	0.27	9.05
82.	26.8	1900	0.9		21.1	2.45	0.30	24.6
83.	15.5	1650	1.5		15.7	2.45	0.21	9.97
84.	43.8	1800	1.5		17.1	2.45	0.24	32.2
85.	43.4	2000	1.5		19.0	2.45	0.27	35.9
86.	511.4	12100	1.8		201.7	0.48	0.20	61.4
87.	196.3	7400	3.0		154.2	0.48	0.20	23.6
88.	57.2	2600	3.0		34.7	0.48	0.20	35.0

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

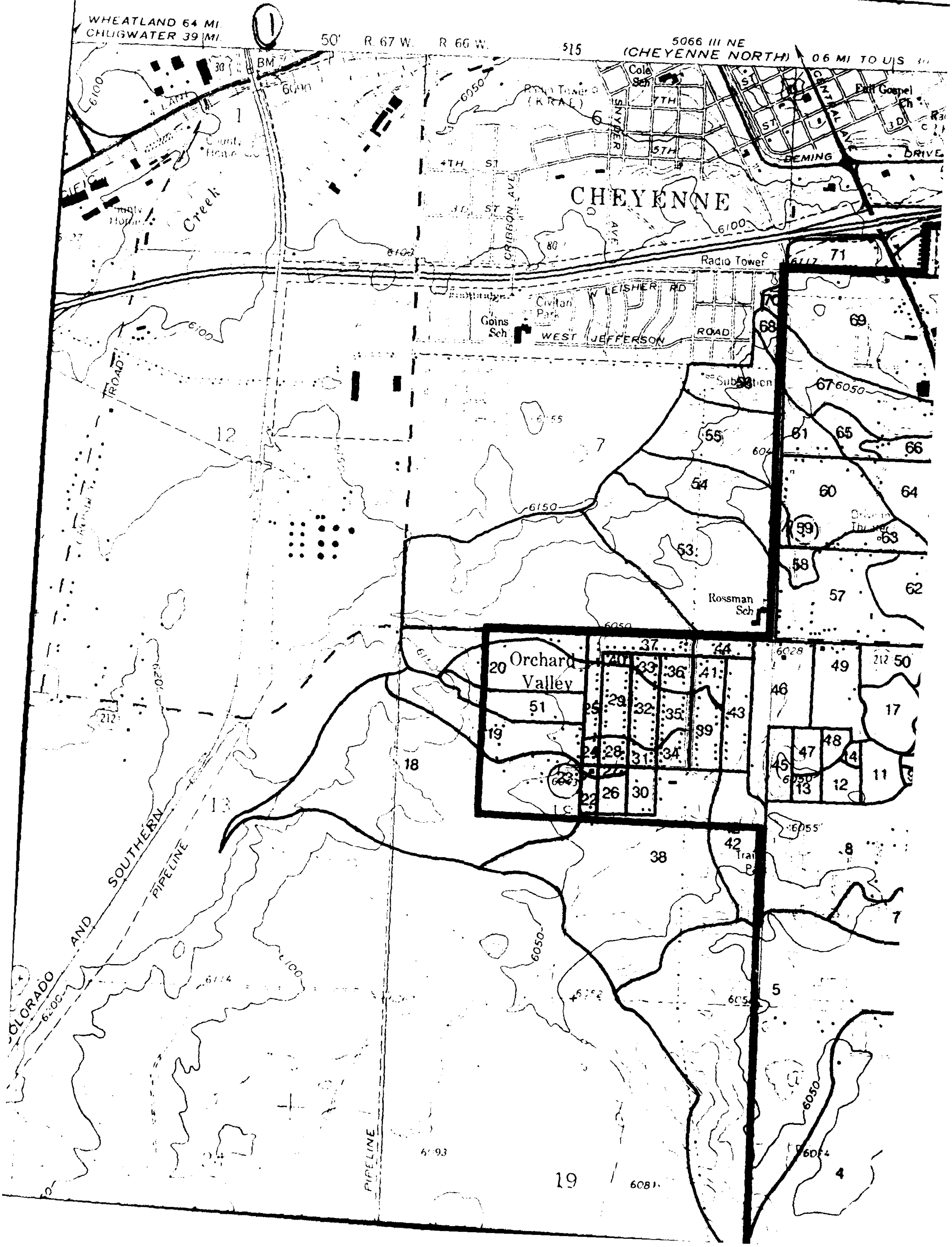
	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
89.	13.7	1100	3.3		6.7	2.45	0.36	15.3
90.	20.0	1900	2.5		12.7	2.45	2.45	24.5

PRE-DEVELOPMENT 100 YR. RUNOFF ESTIMATES

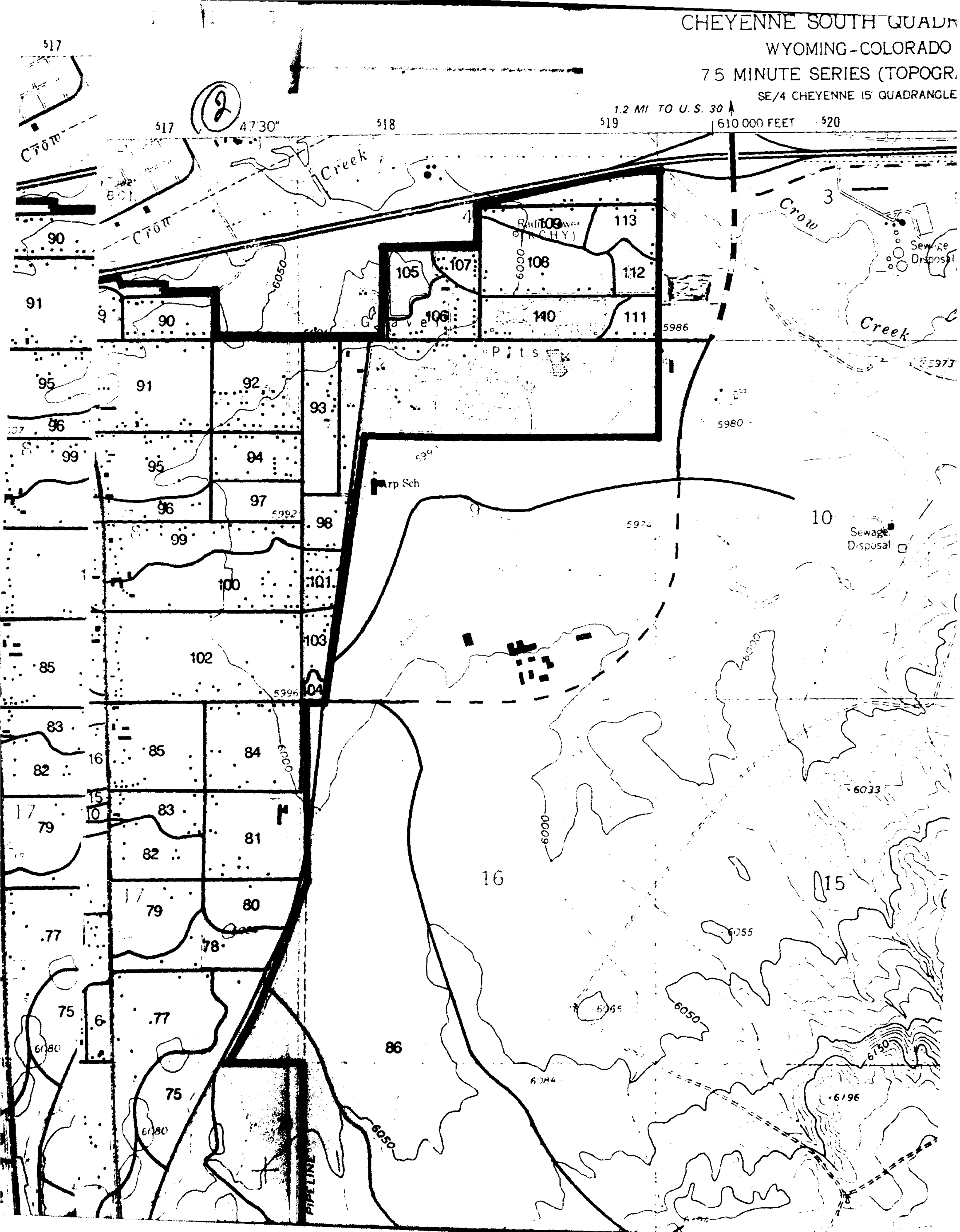
	AREA acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
91.	65.6	3300	2.8		20.8	2.45	0.30	60.27
92.	41.2	2600	1.7		17.5	2.45	0.40	50.47
93.	29.1	2750	0.7		26.2	2.45	0.25	22.3
94.	21.4	1600	0.5		24.2	2.45	0.38	24.9
95.	40.4	2700	1.2		26.5	2.45	0.34	42.1
96.	11.7	2000	0.7		25.6	2.45	0.30	10.7
97.	19.4	1500	0.8		17.9	2.45	0.20	11.88
98.	34.46	2400	0.6		26.7	2.45	0.32	33.8
99.	43.5	2550	1.0		28.3	2.45	0.32	42.6
100.	43.5	3000	0.6		33.3	0.48	0.40	10.4
101.	10.3	900	0.5		13.6	2.45	0.40	12.63
102.	87.	4000	1.0		44.4	0.48	0.34	17.74
103.	10.2	1150	0.5		17.4	2.45	0.25	7.8
104.	4.2	250	1.2		2.6	2.45	0.20	- 2.6
105.	9.1	650	1.25		6.2	2.45	0.20	- 5.57
106.	23.1	1800	0.6		24.0	2.45	0.40	28.3
110.	30.75	1250	2.0		14.9	2.45	0.15	-14.1

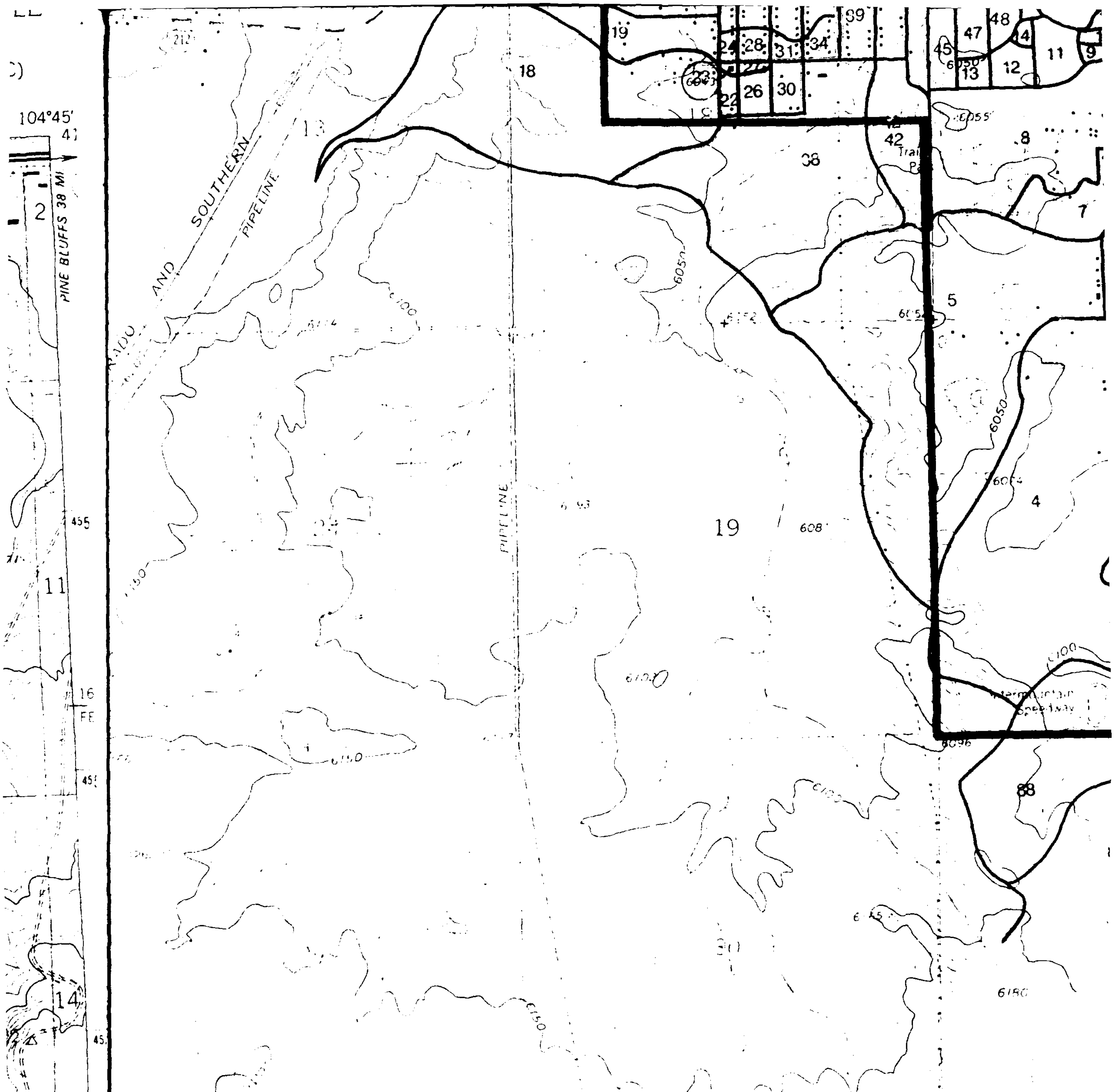
PRE-DEVELOPMENT 100 YEAR RUN-OFF ESTIMATES

AREA	acres	L ft.	S %	V ft/sec	tc min.	I in/hr	C	Q cfs
107.	6.0	1000	1.6		9.31	2.45	0.36	6.6
108.	31.35	1650	1.3		16.17	2.45	0.30	-28.8
109.	19.5	1800	1.0		20.0	2.45	0.30	17.9
113.	15.1	1650	0.6		22.0	2.45	0.32	14.8
111.	10.0	850	0.5		20.2	2.45	0.30	9.2
112.	11.8	850	1.0		9.4	2.45	0.34	12.3



WHEATLAND 64 MI.
CHUGWATER 39 MI.
1
50'
R. 67 W.
R. 66 W.
515
5066 III NE
(CHEYENNE NORTH)
0.6 MI. TO U.S.
BM
6000
Cole Sch
7TH
5TH
4TH ST
3RD ST
Radio Tower (KRAL)
CHEYENNE
SNYDER
CRIBBON AVE
AVE
CENTRAL AVE
DEMING
DRIVE
Full Gospel Ch
County Home
Creek
6100
6050
6150
Radio Tower
W LEISHER RD
Civitan Park
Goins Sch
WEST JEFFERSON ROAD
Substation
Rossman Sch
ROAD
12
7
6
Orchard Valley
6028
212
6055
Trai
COLORADO AND SOUTHERN
PIPELINE
13
18
19
6081
6093
5
4
8
71
70
69
68
67
66
65
64
63
62
61
60
59
58
57
56
55
54
53
51
50
49
48
47
46
45
44
43
42
41
39
38
37
36
35
34
33
32
31
30
29
28
26
25
24
23
22
20
19
17
13
12
11


CHEYENNE SOUTH QUADR
WYOMING-COLORADO
7.5 MINUTE SERIES (TOPOGR
SE/4 CHEYENNE 15' QUADRANGLE
1.2 MI. TO U.S. 30
610 000 FEET
47'30"
Crow Creek
Radio Tower
Gravel Pits
Arp Sch
Sewage Disposal
PIPELINE
16
15
10
3
90
91
92
93
94
95
96
97
98
99
100
101
102
103
104
105
106
107
108
109
111
112
113
140
85
84
83
82
81
80
79
78
77
75
86
6000
6050
6033
6055
6065
6084
6130
6196
6080
5986
5980
5974
5996
5992
5973



SOUTH CHEYENNE PRELIMINARY

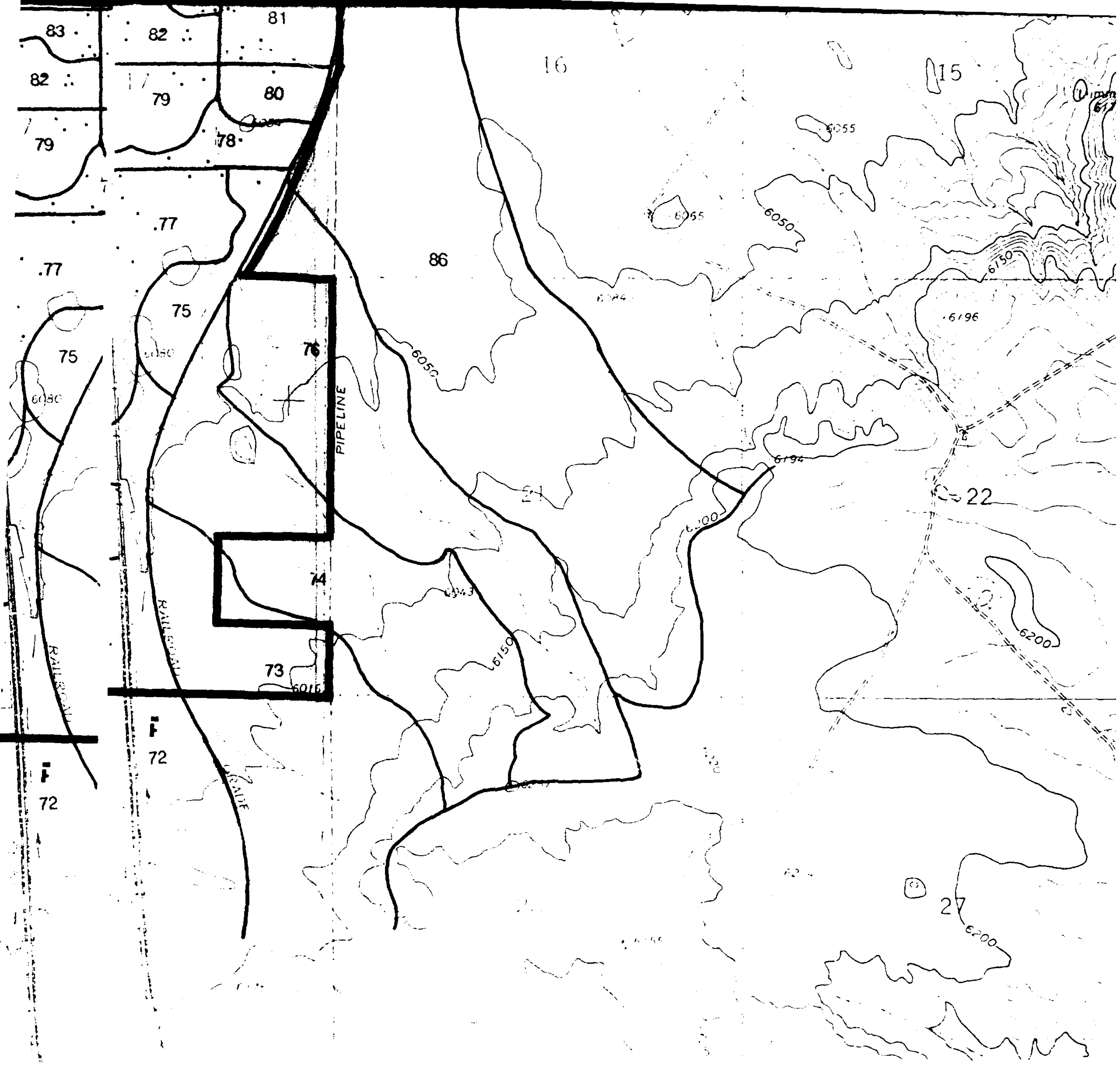
FUNDED THROUGH THE **U.S. DEPT. OF DEFENSE**

BY

LEGEND

STUDY AREA BOUNDARY

DRAINAGE BASIN BOUNDARY



RAIN)RAINAGE PLAN

.01 PR(801 PROGRAM

VI,p.c .VI,p.c.

00 WES 00 WESTLAND RD.

EYENNI IEYENNE, WYO. 82001

A